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ASSESSMENT OF SOIL-STRUCTURE-INTERACTION EFFECTS ON THE DYNAMIC RESPONSE OF STEEL HIGH-RISE MOMENT RESISTING BUILDINGS

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ABSTRACT

The dynamic response of structural systems under earthquake ground motions is significantly affected by the flexibility and damping of the soil-foundation system, either for shallow (rafts) or deep (pile) foundations. Recent numerical and experimental tests carried out world-wide have shown that high-rise buildings, which are most commonly built employing as steel lateral resisting systems, e.g. perimeter moment resisting frames (MRFs) and concentrically (CBFs) or eccentrically (EBFs) braced frames, may also be influenced by soil-foundation-structure interaction (SFSI) under specific design scenarios. Nevertheless, few studies address the seismic performance of steel high-rise buildings including SFSI. The present analytical work focuses on the earthquake response of a steel framed structure designed in the framework of the SAC Steel Project. The twenty-storey MRF was selected and assessed with and without considering the role of SFSI. The foundation of the sample frame was designed for various conditions of soil flexibility; the design was performed in compliance with European and international seismic standards. An extensive parametric analysis was carried out and the variations of the fundamental periods of vibrations along with higher modes participation were assessed. It was found that the dynamic response of the sample building is indeed affected by the design of the foundation, the relative flexibility of the foundation-soil system as well as the structural modelling employed. The proposed case study is intentionally examined on the basis of simple dynamic characteristics (i.e. modal contribution parameters) in order to highlight their dependency on easy-to-derive foundation-soil properties. Although the approach is inevitably case-dependent and as such, the fundamental conclusions drawn cannot be easily generalized, it is a practical attempt to quantify in a simplified manner the effects of SFSI on the seismic response of tall buildings.

Keywords: high–rise buildings, steel buildings, soil-structure interaction

INTRODUCTION

Tall buildings are being built in densely populated areas world-wide. Such buildings are often located in regions with high seismic and/or wind hazards. For instance, several high-rise structures exist in the California, in Japan, and many others are under construction especially in China, Taiwan and in several cities in the Middle East. Nevertheless, tall buildings are still one of the few constructed facilities which exhibit complex dynamic response that often requires the implementation of expensive scaled model testing or advanced health monitoring techniques.

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The lateral resisting systems of high-rise buildings in particular, generally employ structural steel and/or composite steel and concrete technologies. The most common structural configurations encompass either moment resisting frames (MRFs) or braced frames (BFs). For such frames, the first mode does not necessarily dominate the dynamic response and a large number of modes may be required in order to activate a significant portion of the modal mass.

The potential modifications of the modal participation factors (MPFs), which are essentially the measure of the degree to which the *nth* mode participates in the response, are important because they can in turn affect the dynamic and seismic response of the building at each time step *r(t)*, as this results from the combination of the individual response contributions of all modes:

$$r(t) = \sum_{n=1}^{N} r_n^{st} \left[\omega_n^2 D_n(t) \right] \qquad (1)$$

where N is the number of modes, r_n^{st} denotes the modal static response due to external forces S_n of the particular mode n, ω_n is the natural frequency of the n th mode and $D_n(t)$ is the displacement of the n th mode SDOF system.

It is also notable that higher modes can affect the dynamic response of tall buildings under both earthquake and extreme wind loading. In fact, the subsequent multiple point surface pressure that results from wind loading also depends on the modal shapes while potential modal coupling is deemed to significantly amplify wind load effects (Zhou et al., 2002).

The above discussion points out that the identification of modal participation factors is of paramount importance in the analysis and design of high-rise MRF steel buildings. The assessment of these factors is rarely based merely on analytical approaches. Commonly the modal participation factors of the building are derived indirectly using monitoring procedures due to ambient vibrations (Li et al., 2002, Pan et al., 2004), strong wind excitations (Li, and Wu, 2004), earthquake excitation (Brownjohn and Pan, 2001, Celebi, 2006) or combined Finite Element model updating based on various monitoring techniques (Ventura et al., 2005).

Predicting the modal characteristics of high-rise buildings though, is inevitably related to soil and foundation compliance. Conceptually, it may be assumed that soil-foundation-structure interaction (SFSI) affects also the base shear demand through inertial and kinematic interaction: the first is related to the oscillation of the superstructure and the subsequent building-induced foundation translation and rocking deformations which essentially lead to structural period elongation. The latter can be approximately quantified in terms of additional structural damping at the foundation-soil interface (Mylonakis & Gazetas, 2000; Stewart et al, 2002).

Unfortunately, the role of SFSI effects on the overall response of steel high-rise buildings has not yet been investigated in detail and hence, in construction and design practice, soil is considered primarily as a source of building settlement independently of the actual seismicity of the region (i.e. Baker et al., 2004, Katzenbach et al., 2005).

The reasons that SFSI is often completely ignored in the design process of high-rise buildings may be possibly related to the prevailing perception that:

a) The inherent complexity of the problem (in terms of modeling soil properties, the foundation and the soil-foundation interface) can have higher uncertainty than neglecting the problem completely and relying on other safety factors, especially in the light of the already complex dynamic response of tall buildings.
b) Steel high-rise buildings are usually regular systems with simple or double symmetry hence they are assumed as less prone to torsional excitations due to higher modes of vibration.
c) The period elongation that is anticipated due to the activation of soil flexibility will normally lead to reduced levels of seismic forces, especially if the building is high. It is notable that this

is also prescribed by the NEHRP Recommented Provisions for Seismic Regulations for New Buildings and other Structures (BSSC, 2001) for structures with fundamental period that exceeds 0.5sec. Based on this approach but also on a large set of field data, SFSI effects are found to be negligible in case of long-period structures (Stewart et al., 2003) because of their relatively high lateral deformability.

d) Higher modes are for years assumed as unaffected by SFSI effects (Jennings and Bielak, 1973), hence only fundamental mode structural parameters are assumed to be required for design and the calculations can be sorely based on base shear estimation.
e) Additional damping is expected to be developed at the soil-foundation interface, thus limiting the energy that is transmitted to the superstructure. As a result, ignoring the SFSI-induced damping is considered a conservative approach.

On the other hand, other studies have shown that modal coupling and participation is difficult to be assessed in the case of high-rise buildings. Safak & Celebi (1991) analyzed extensively the recorded responses of two high-rise buildings in San Francisco during the 1989 Loma Prieta (California) earthquake where SFSI effects induced by the soft soil conditions were experimentally identified. Cross spectra among horizontal ground level motions and those of the upper floors were investigated and revealed particular soil-dependent resonance. A similar observation was also made by Muria – Vila et al. (2004) who studied experimentally and numerically the seismic response of two high-rise buildings recorded during the 1985 earthquake in Mexico City.

Moreover, specifically for MRFs, it has been shown that not only higher modes are important but the percentage of their contribution in all response components increases with the number of storeys (Daneshijoo & Gerami, 2003) thus leading to more non-uniform distribution in height of the drift demands even if soil conditions have not been accounted for. This importance of higher mode effects in steel high-rise MRFs has also been verified by Santa-Ana and Miranda (2000).

The scope of this paper therefore, is to study the simplest possible, yet adequately realistic, MRF high-rise structure with a plan layout of double symmetry. Through a set of parametric modal analysis, the modification of its dynamic characteristics for various combinations of foundation configurations and soil conditions is evaluated. Clearly, the response of the building in the time domain (which is not studied herein) would provide a more thorough insight on the actual effect of SFSI. However, the question set is intentionally focusing on the single aspect of the system's dynamic characteristics in order to potentially shed light on whether the inclusion of soil flexibility indeed affects the dynamic properties of tall buildings. The assessed structure is briefly illustrated in the following paragraph.

THE ASSESSED MOMENT RESISTING FRAME

The moment resisting frame (MRF) assessed in the present study was initially designed in the framework of the FEMA/SAC program (SAC Joint Venture, 1995). This program was aimed at estimating the uncertainty, variability and bias associated with the prediction of interstory drift demand of steel buildings utilizing various commonly used analysis techniques. In the SAC Steel Project, a set of 9 model buildings, representing 3-story, 9-story, and 20-story structures were designed by three consulting firms for three cities with different seismic hazard in the United States: Boston (Massachusetts), Seattle (Washington) and Los Angeles (California). These city are, in fact, located in regions of low, moderate and high seismicity.

In the present analytical study, the set of above buildings is deemed ideal for performing the foreseen investigation, because their seismic performance has been extensively studied during the last decade (e.g., Gupta and Krawinkler, 1998-a; Gupta and Krawinkler 1998-b; Maison and Bonowitz, 1999; Krawinkler, 2000; Gupta and Krawinkler, 2000-a; Gupta and Krawinkler, 2000-b; Cornell et al., 2002, Di Sarno, 2002; Mele et al., 2004 and Krishnan et al., 2006; among many others). Moreover, recent analytical work by Goel and Chopra (2004) has demonstrated that higher modes are important for such structures.

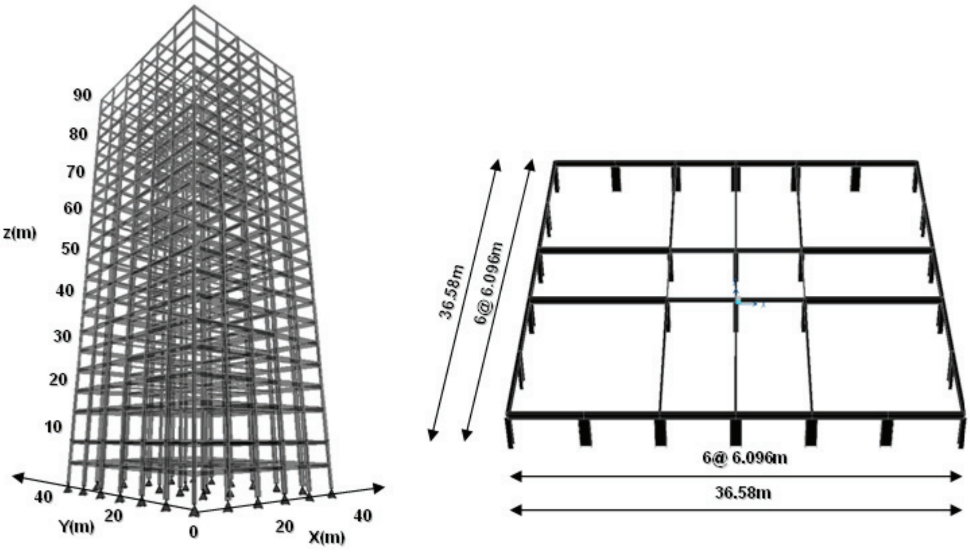


Figure 1. – 3D and plan layout of the assessed building in Boston

From the above set of nine buildings, the 20-storey high-rise office building located in Boston is adopted as sample structure for the present study. As shown in Figure 1, the MRF building has a rectangular floor plan 30.5 x 36.6 m, with five and six 6.10 m bays respectively, along the principal directions. Inter-storey height is 5.49 m at the first storey and 3.96 m at all other storeys, whilst the basement level is 3.66 m high. Four MRFs are located along the perimeter of the building to provide lateral and torsional resistance. Perimeter systems consist of four bays with strong axis columns rigidly connected to the girders plus an external bay where the girder is shear-connected to the corner columns, such that biaxial bending is prevented in all building corner columns. Beam span loads (dead and live) are equal to 14.88 kN/m and joint vertical loads are equal to 158 kN and 107 kN at interior and perimeter joints, respectively. Beam loads of 12.65 kN/m, while joint vertical loads of 140 kN (interior joints) and 92 kN (exterior joints) were used for the roof. The slab is a floor system of 76.2 mm metal deck with 63.5mm normal weight concrete fill. The cross sections employed for beams and columns can be found in Di Sarno (2002); they are typical wide flange profiles used for steel framed structures. A nominal yield strength of 345 MPa is adopted for both girders and columns of all perimeter frames.

Structural Modelling and Dynamic Properties

For the buildings analysed in this study, finite-element models were developed based upon careful reference to the design drawings and using currently available commercial software SAP2000 (Computers and Structures Inc, 2002), a finite element program for elastic and inelastic (static and dynamic) analyses. The frame skeleton employs centreline dimensions and was modelled by means of three-dimensional elastic beam-column elements, whilst rigid diaphragms were used at each storey level for the composite. Geometric non-linearities, i.e. P-Δ effects, were included within the analyses, hence eigenvalues were computed with regard to elastic and geometric matrices. The total mass of the 20-storey buildings is 12,766,349 kg; such mass includes total dead and live loads. Thus, the average building density is 130 kg/m^3. The eigenvalue analyses provided the translational and torsional frequencies summarised in the next section along with the participation mass ratios relative to the fundamental translational modes along orthogonal directions.

ALTERNATIVE FOUNDATION AND SOIL CONDITIONS

In order to investigate the effects of SFSI on the response of the sample steel high-rise building, seven alternative finite element models were developed. More specifically, six different types of foundation for this building have been preliminary designed and modeled (B1, B2, B31, B32, B33 and B34) and their dynamic properties are compared with those of the fully fixed building (Model A) that is assumed as the reference structure (benchmark model).

The six alternative foundation conditions correspond to the case of a basement of 3.50m height supported on soft (Model B2) or relatively stiffer (Model B1) soil formations. The basement is modeled using a set of 2-D (shell) finite elements at a grid of 1.25x1.25m and is assumed to be made of reinforced concrete. Both the lateral and vertical springs, which were used in order to simulate the soil flexibility were derived from the formulation suggested by Mylonakis et al (2002) with the assumption of having a large scale, embedded, surface foundation. It is noted that since high-rise buildings are most commonly supported on pile foundations, the aforementioned models (B1 and B2) are essentially an upper bound of the flexibility of the soil-foundation system. Models B31, B32, B33 and B34 on the other hand, represent four more realistic cases for the Boston MRF building. For these models, a pile group is adopted and designed for the foundation, while the basement is retained as part of the structure that acts as a massive piles cap. The diameter of the piles has been kept identical for comparison purposes (i.e. 0.80 m) but the length L, and the pile spacing S is determined depending on the surrounding soil stiffness. The corresponding spring coefficients are calculated based on the equations suggested by Makris & Gazetas (1992). It is noted that the increased pile group flexibility due to pile-to-pile interaction was accounted in a simplified manner with the use of additional springs among the piles. The main features of the seven alternative FE models are summarized in Table 1.

COMPARATIVE RESPONSE OF THE ALTERNATIVE SOIL-FOUNDATION-MRFS STUDIED

For the models presented above, a series of modal analyses is performed in order to identify the corresponding dynamic characteristics of the soil-foundation-superstructure system. At least 20 modes where considered important in the analysis. However, the periods of the first 6 modes are presented in Figure 2 for comparison. It is notable that the computed fundamental period of the fixed base system is equal to 3.52 sec, that is, almost 50% higher than that predicted by the approximate formulas available in the literature or provided by seismic codes. In particular, according to FEMA273 (1997) the fundamental period T (in seconds) of the building is approximated through the following equation:

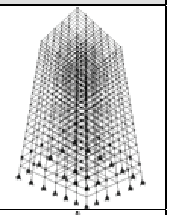
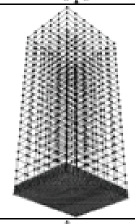
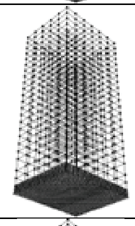
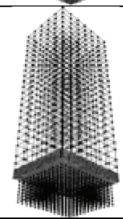
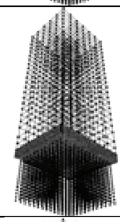
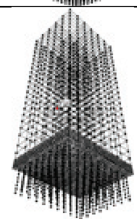
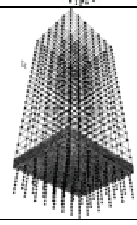
$$\mathrm{T} = C_t \cdot h_n^{3/4} = 2.45 \text{ sec} \tag{2}$$

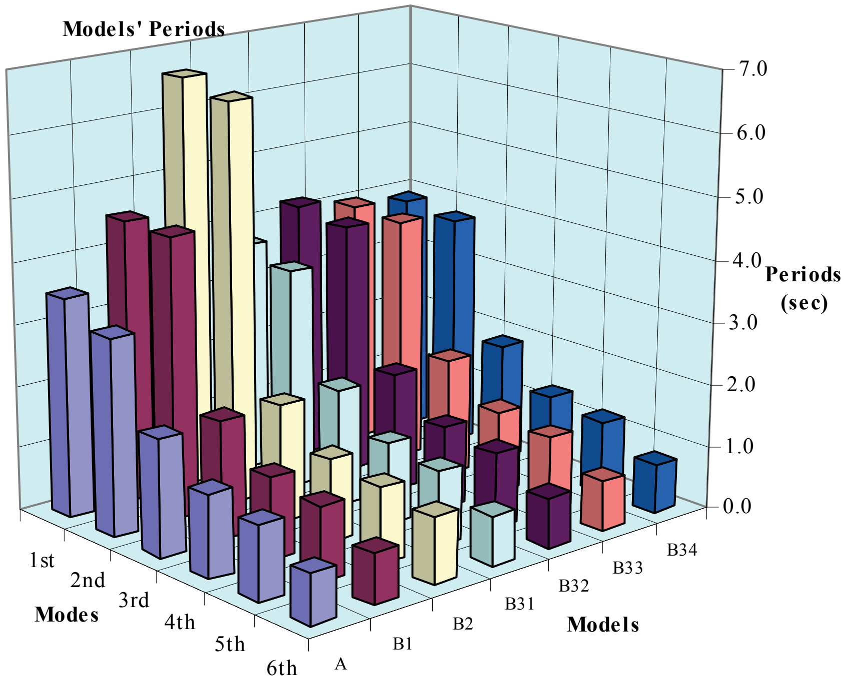
where C_t is equal to 0.035 for moment resisting frame systems of steel, and h is equal to 290 feet. The same value is derived with the similar expression provided by Eurocode 8 – Part 3 (CEN, 2004). The discrepancy between the period computed using modal analysis (i.e. 3.52 sec) and that derived from the simplified code formula is due to the fact that, codes tend to provide a conservative (higher) value of structural stiffness (i.e. leading to higher seismic forces) in order to account for the effect of infills, partitions and any other source of non-structural components. These elements increase the lateral stiffness of framed buildings, particularly steel ones. Indeed, based on measured data of 42 steel moment resisting frames in California, the best-fit 1-σ curves yield an expected fundamental period in the range of (Goel and Chorpa, 1997a):

$$\mathrm{T} = 0.028 \cdot H^{0.8} = 2.61 \text{ sec (1-σ : lower bound for displacements)} \tag{3}$$
$$\mathrm{T} = 0.035 \cdot H^{0.8} = 3.26 \text{ sec (best fit : average expected displacements)} \tag{4}$$
$$\mathrm{T} = 0.045 \cdot H^{0.8} = 4.19 \text{ sec (1+σ : upper bound for displacements)} \tag{5}$$

As a result, the computer herein eigenfrequencies are deemed reasonable. Moreover, in the present study, the fundamental period and all the other modal components, are not considered in absolute terms but on a relative basis among the building with the various foundation configurations and the corresponding fully fixed one.

Table 1. Alternative Soil-Foundation-Structure FE design assumptions and modeling features.

Model	Foundation Characteristics	Soil	FE Model
A **(benchmark)**	Fully fixed	Fixed	
B_1	Only basement	E_s = 20.000 kPa	
B_2	Only basement	E_s = 5.000 kPa	
B_{31}	Basement & Piles (dense piles) ⇨s/D = 2.45/0.8 = 3.06 ⇨L = 10.50 m	E_s = 20.000 kPa	
B_{32}	Basement & Piles (dense piles) ⇨s/D = 2.45/0.8 = 3.06 ⇨L = 20.0 m	E_s = 5.000 kPa	
B_{33}	Basement & Piles (moderate - dense piles) ⇨s/D = 3.65/0.8 = 4.56 ⇨L = 10.50 m	E_s = 10.000 kPa	
B_{34}	Basement & Piles (sparse piles) ⇨s/D = 5.45/0.8 = 6.81 ⇨L = 10.50 m	E_s = 20.000 kPa	

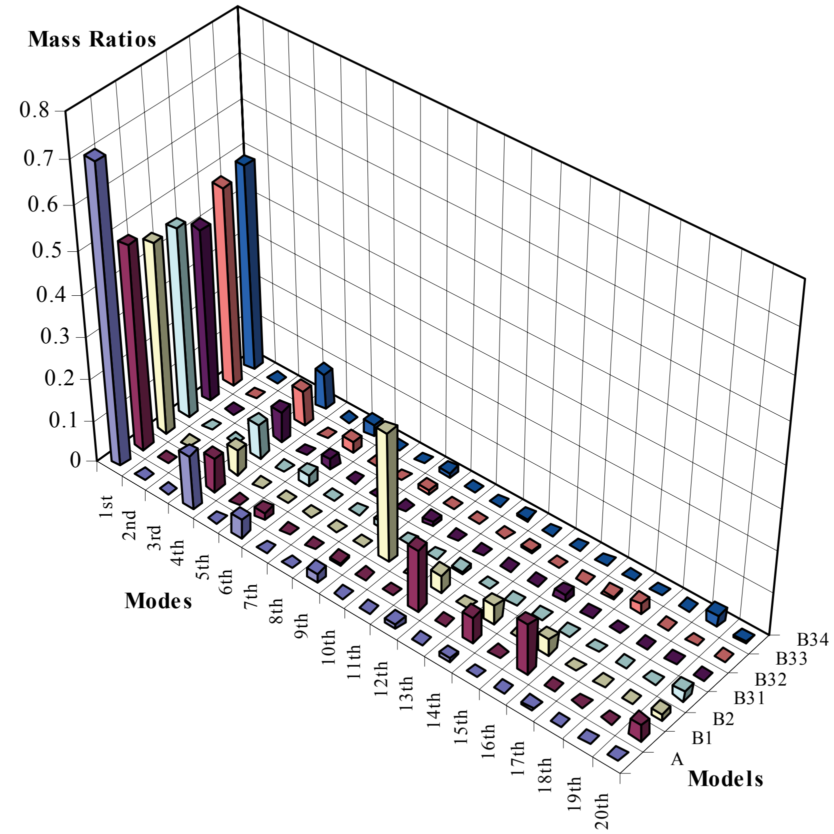


	1st	2nd	3rd	4th	5th	6th
A	3.52	3.17	1.88	1.32	1.19	0.80
B1	4.55	4.54	1.89	1.32	1.19	0.80
B2	6.64	6.46	1.90	1.33	1.20	1.07
B31	3.75	3.56	1.89	1.32	1.19	0.80
B32	4.20	4.07	1.89	1.32	1.20	0.80
B33	4.00	3.96	1.89	1.32	1.19	0.80
B34	3.90	3.78	1.89	1.32	1.19	0.80

Figure 2. Periods of the first six modes of the analysed finite element models

It is also noted that, as described previously, Model B2 is an upper bound of SFS system flexibility since a deep foundation is not adopted despite the relatively soft soil conditions. As a result, excluding the particular B2 model, the fundamental period elongation is of the order of 7-20% for the cases that a pile foundation was adopted, and as anticipated, larger (30%) for the case of basement-only foundation on moderate soil. For the extreme case of Model B2, the computed period lengthening is approximately 90%.

The mass ratios on the other hand, which are found to be activated by the first twenty modes in each of the abovementioned FE models of the sample building, are shown in Figures 3-5 corresponding to mass ratio activation along X, Y and around Z axis respectively. The computed values demonstrate that even for the fixed base model (A), the effects of higher modes are significant, since the modal participation mass ratios are less than 70%, in both X and Y directions. This is of course anticipated for the case of high-rise buildings, as well as for the specific buildings and has been pointed out already in the literature (Goel and Chopra, 2004).



All Models
Mode 4

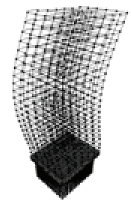
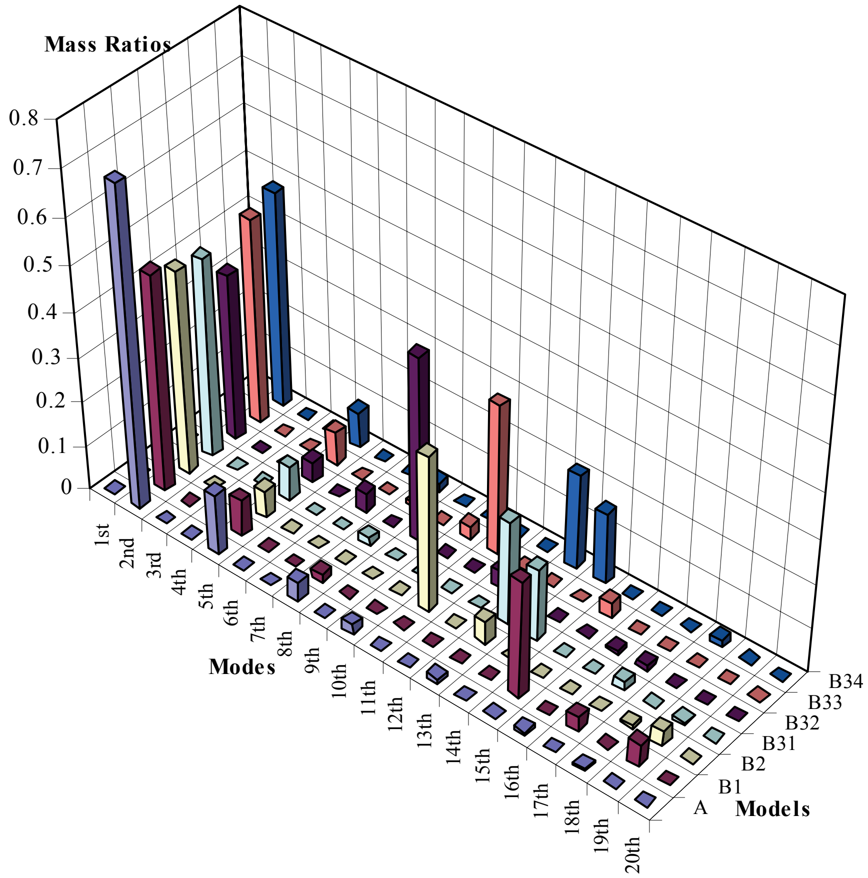
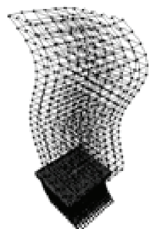


Figure 3. Modal Participating Mass Ratios (Ux)



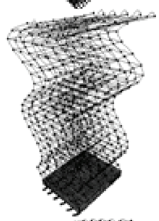
Model B32
Mode 9



Model B31
Mode 13

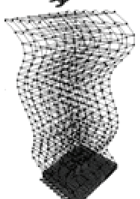
Model B34
Mode 13

Model B34
Mode 14



Model B31
Mode 14

Model B33
Mode 11



Model B1
Mode 15

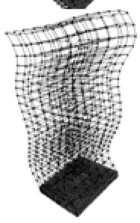


Figure 4. Modal Participating Mass Ratios (Uy)

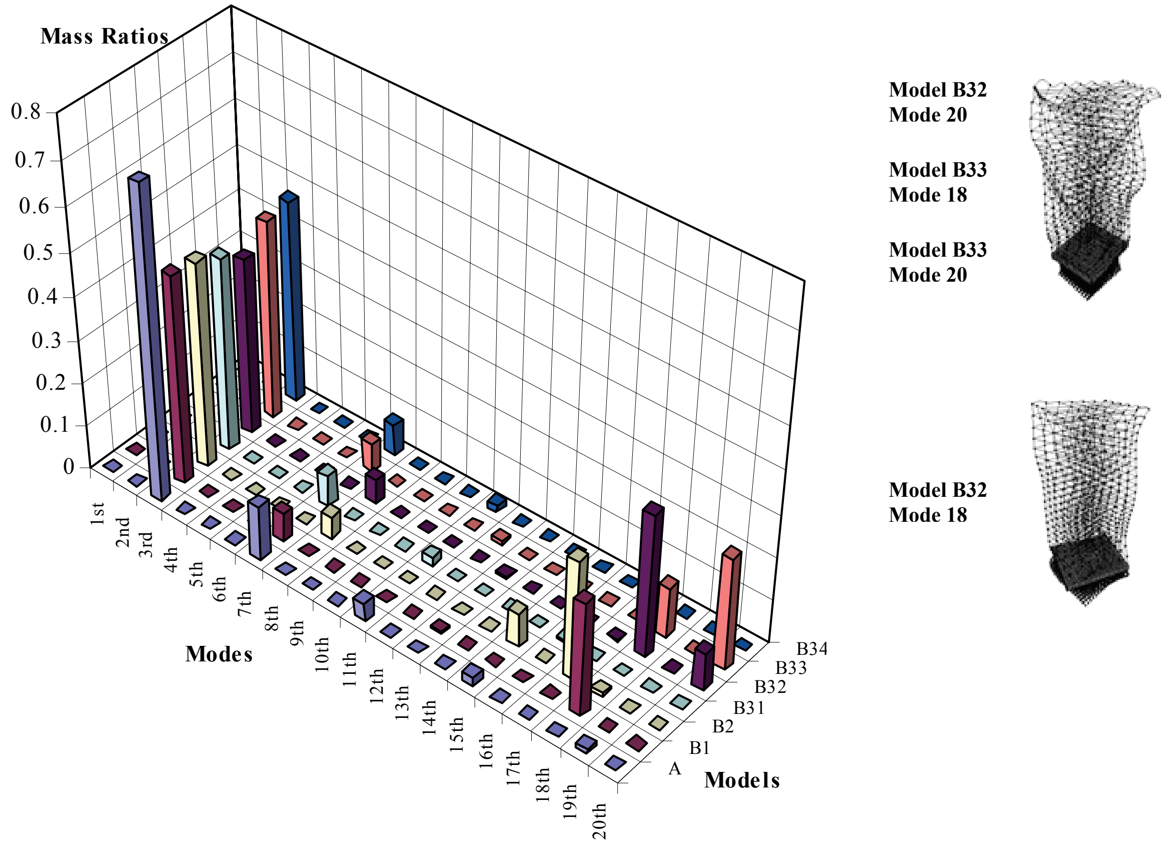


Figure 5. Modal Participating Mass Ratios (Rz)

What is of particular interest though, is that, as displayed in Figures 3 to 5, the contribution of the first mode in each direction is significantly reduced when soil and foundation flexibility is accounted for, a reduction that is related to all first three modes. The participation of the first mode to the U_x direction is reduced from 69% to 40-46%, the participation of the first mode to the U_y direction is reduced from 63% to 28-42%, while participation of the first mode around the R_z axis is reduced from 58% to 28-44%. This uniform average 50% reduction to the contribution of the first three modes is clearly observed since they are all found to be almost completely uncoupled. Another interesting issue is that there is no significant discrepancy between the participation computed for the all the flexibly supported versions of the building under study. In other words, considering soil and foundation stiffness (as a whole) during the modal analysis of the particular MRF structure affects its dynamic characteristics more significantly than the assumptions made by the designer regarding the stiffness of the soil itself.

Furthermore, in all three directions, it is seen that soil compliance leads to higher modes triggering. These modes are illustrated also in Figures 3 to 5 where it is seen that their contribution is not negligible as it can even reach the order of 30% of the total mass. This can be of particular significance since most of the higher modes observed had only minor (i.e. less than 3%) contribution to the dynamic response of the fixed-base structure. The latter also applies to torsional modes which are additionally activated by soil flexibility despite the double symmetry of the building – an observation that is in agreement with the results of a parametric study (Wu et al., 2001) showing that the increasing height-to-base ratio of a structure generally amplifies its torsional response.

Despite the clarity of the results presented above though, it is important to avoid generalizing the findings. The main reason for such reluctance is that soil-foundation-structure interaction is a very complex and multi-parametric phenomenon that has to be seen particularly from a dynamic point of view, since it is well known that the dynamic impedance at the soil-foundation interface can be strongly frequency dependent. As a result, additional damping is expected to be developed both at the soil-foundation interface as well as on the superstructure (due to structural yielding), thus affecting the overall SFS response of the system in the time domain. Moreover, even in terms of eigenvalue analysis, it is not obvious that all structural modes would be equally damped, especially the higher modes activated by the compliance of the soil, while for multi-storey buildings the inelastic periods may differ substantially from the elastic ones based on the various sources of energy absorption of the superstructure (Elnashai and Mwafy, 2002). It can thus be argued that an exact knowledge of damping characteristics of tall buildings may be essential to the prediction of their dynamic response. It is foreseen therefore that the present study will be extended by accounting for the above reservations in order to attempt to extrapolate the observations and conclusions drawn for the study of the particular MRF building.

CONCLUSIONS

The present analytical work focuses on the dynamic characteristics of a 20-storey steel framed structure designed for a region of low seismicity within the framework of the SAC Steel Project. The sample structures was assessed with and without considering the role of soil-foundation-structure interaction (SFSI). The results of an extensive parametric analysis have shown that the fundamental period of vibration and higher modes participation of the high-rise frame are indeed strongly affected by the design of the foundation, the relative flexibility of the foundation-soil system and the properties of the finite element models utilized in the analyses perfomed. Despite the fact that the conclusions drawn cannot be easily generalized, the case study is an attempt to quantify the effects of SFSI on the complex dynamics and seismic response of tall buildings. Further analytical work is required to investigate the effects of SFSI on high-rise buildings located in areas with different seismic and/or wind hazard.

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